STRUCTURAL INVESTIGATION
OF
79 ELM STREET
HARTFORD, CONNECTICUT

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PREPARED FOR:

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INTRODUCTION

The Department of Environmental Protection (DEP) Annex is an 80-foot wide by 215-foot deep addition, 8 stories tall, completed five years ago. The basement and 3 floors of concrete support a structural steel frame above.

On two occasions, during normal business hours, a loud, single noise occurred without warning. This noise was alarming to the personnel working near the source and was heard and felt two stories above. The source on January 4, 1999 at 11:45 a.m. was at column I18 between the 5th and 6th floor. On December 23, 1998 at 9:30 a.m., the source was at column P17 between the 2nd and 3rd floor.

On both occasions, the noise source came from the intersection of floor beams and a column, just before the floor. The investigation of this was complicated by the presence of cementitious fire protection which has been sprayed on the steel. This material was removed to allow inspection. It was further complicated by a desire to not interfere with the daily work of DEP staff.

Cracking of concrete block in the stair tower has been reported. This was considered as it might be related to the noise.

Gibble Norden Champion was authorized by the State of Connecticut Department of Public Works to perform the following scope of services:

1. Review the required design and construction records and drawings and conduct such interviews and discussions with involved parties as necessary.

2. Visually inspect the entire building for evidence of the source of the noises and movement experienced and/or other signs of distress.

3. Perform such measurements as are necessary to ascertain any suspected displacement not observable by visual means.

4. Confirm by measurement and analysis the sizes and capacities of the structural elements at the suspected point sources of the noises experienced.

5. Prepare written reports (DRAFT and FINAL) containing a complete record of findings, recommendations and opinions and present at a meeting of appropriate parties.

INVESTIGATION METHODS USED

First we listed the possible causes of structural distress to the building.

Then we determined an appropriate means of evaluating each possible cause.

We collected data by the following methods:

- Review of pertinent design and construction records.
- Interviews with people associated with the design, construction, occupants and unassociated knowledgeable people.
- Observations of the building particularly the two steel intersections in question and the two stair towers.
- Measurement of the floor levelness.
- Measurement of all steel member sizes at the two steel intersections.

We analyzed these observations, and we calculated the typical floor steel member sizes to confirm the original design.

POSSIBLE CAUSES OF STRUCTURAL DISTRESS

POSSIBLE CAUSE	STRUCTURAL SYSTEM	METHOD OF INVESTIGATION
1. Differential settlement of foundation.	60" and 42" diameter concrete caissons, 65 feet deep (plus or minus), bearing on rock.	a. Measurement of floor levelness at 3rd floor using a laser level. b. Observations of masonry cracking.
2. Failure of welds at beam to column connection.	4½" concrete on 3" metal deck, total 7½". Support by 14" and 16" deep composite steel beams and 24" deep steel girders running north-south.	a. Visual inspection of welds at two identified beam to column intersections. This necessitated removal of the sprayed on fire protection.
3. Failure of bolts at beam to column connection.	Same as 2.	a. Visual inspection of bolts at same two locations.
4. Failure of column splices	10" deep steel columns with sprayed fire protection.	a. Review shop drawings to see if column splices are near sources of noise.
5. Under design of steel beams, girders or columns.	Same as 2.	a. Calculate required sizes.
6. Substitution of undersize beams, girders or columns.	Same as 2.	a. Check sizes on Engineer's drawings against shop drawings. b. Check field measured size against shop drawings.
7. Excessive lateral movement.	The basement and two levels of parking are reinforced concrete. From the top of the second floor (first office area) to the roof is a structural steel frame.	a. Observation of masonry cracking. b. Computer analysis of moment resisting frame.

FINDINGS

1. Differential Settlement:

   Measurements of the 3rd floor surface at each column showed some variation of levelness. However, these were within construction tolerances and therefore do not indicate foundation settlement. See Appendix A for readings.

   The measurements were made at columns to avoid confusing the readings with beam deflections. At column I18 we found a high point. We then checked that location on the 2nd floor which showed that line I is level. The high point at the 3rd floor is due to concrete finishing variation and have no material consequences on the issue under consideration.

   Observation of interior and exterior masonry walls does not show cracking patterns associated with differential settlement.

2. Failure of Welds:

   At the locations where fire protection was removed, the welds were observed to be the correct size, length and acceptable quality. No cracking was seen. See Appendix B.

3. Failure of Bolts:

   At the locations where fire protection was removed, the bolts were observed to be the correct size, type and acceptable condition. See Appendix B.

4. Failure of Column Splice:

   From a review of shop drawings, the location of column splices were not located at the source of noise on the two recent events. The type of splice was direct bearing of the column above and below with both bearing surfaces milled square. This seems unlikely to be a source of noise.

5. Under Design of Steel:

We have done calculations to confirm the steel members were properly sized. Stephen Carey of the Department of Public Safety confirmed that a design review of the structure was done before construction started by Dave Raffles under contract to DPW. The design of the structure was found acceptable by him.

6. Substitution of Undersize Members:

We compared the member sizes on the engineering drawings with the shop drawings and found no differences.

We field measured every dimension of all beams, girders, columns and connection pieces at the two exposed locations. These were compared to shop drawings and the AISC Steel Manual. As expected we found small dimensional variations but determined that all member sizes measured were the correct sizes.

7. Excessive Lateral Movement:

Our observation of masonry cracking showed a repetitive horizontal crack at the same locations in both the north and south interior stair towers. The cracks were predominantly in or near the first masonry joint below each floor. The cracks are visible in all four interior walls of the stair towers. The average crack measured was 0.060" wide by 0.75" deep.

Below the 2nd floor, at both parking levels and in the basement, this crack pattern did not exist.

Some diagonal cracking was evident in the stairs, radiating from the corner of doors. These were not extensive or unusual for concrete masonry walls.

No significant patterns of structural cracking were noted on the exterior walls.

We performed a computer analysis of the structural frame in the short direction. The lateral load resisting system in this direction consists of small steel plates welded to the floor beams on the column lines and to the column flanges. See Appendix B.

This type of moment connection is intended to resist wind against the building but to be flexible enough to not resist gravity induced moments. This type of connection was popular in Connecticut before seismic design became a requirement in the October 16, 1989 Code update.

Our analysis started at the second floor and went up to the roof. We did not include the concrete floors and columns because they are much stiffer than the steel system. This analysis is a good indicator in predicating the approximate magnitude of side sway. The wind moment connections used are flexible and allow more rotation than the fixity assumed in the program, therefore larger lateral movements are possible. The software also ignores the beneficial effect of interior walls that may reduce the movement.

We calculated a lateral movement at the roof, when the Building Code design wind blows from the west of 4¼" if the bottom of steel columns are pinned, and 2¾" if they are fixed.

There is no mandated limit, but the industry standard is to limit lateral movement to the height divided by 500. For the steel portion of this building that would be H/500 = 0.002. The calculated movement for comparison is:

Pinned base:	0.005
Fixed base:	0.003

The correct value is probably between these two numbers. What this tells us is this is a flexible building having large horizontal movements when subjected to wind.

Wind is resisted in the long direction (north/south) by shear walls. We have obtained weather records to see what the wind velocity and direction was at the time of the two events. In Appendix C are the hourly records at Windsor Locks. They show wind from the west, but velocities lower than the maximum the building has experienced on other dates.

8. Special Inspection:

Special Inspection/Testing Laboratory reports were obtained from Smith Edwards Architects, Architect of Record, for the design of the Annex. A review of the reports indicated a thorough review of the steel erected in the field. A small amount of inspection took place in the shop during fabrication.

In general any discrepancies found appear to have been resolved and re-inspected. Some reports were found regarding the specific members, within the connections in question, but no problems were noted. Non-destructive testing was done on many moment connections, but we could not find reports which pertained to the specific connections being reviewed. The information available to the testing laboratory at the time of fabrication and construction allowed them to attest to the building's compliance to Building Code requirements. See Appendix D.

INTERVIEWS

Robert Disque, P.E. He was the Technical Director of AISC (American Institute of Steel Construction) for many years and an internationally recognized expert on steel connections.

The phenomenon known as "banging bolts" was first reported to AISC Headquarters in the early 1980's. It has been reported to AISC by building owners and structural engineers several times a year since then. It is a noise that has been described as sounding like a rifle shot. A member of the Gibble Norden Champion staff has actually heard this noise in a structural laboratory and agrees.

Although no one has been able to determine for certain the cause of the noise, AISC and knowledgeable structural engineers are fairly confident that the noise is caused by fully tightened bolts, in a shear connection, slipping into bearing under load. It should be noted that the load at which the slip would take place is far below the design load prescribed by code.

In 1992, the ASCE Committee on the Design of Steel Building Structures studied the problem and wrote a report in the Journal of Structural Engineering, Vol. 118, No. 12, December 1992. A copy of the report is attached. The final paragraph states:

> The banging bolts question is strictly a serviceability issue. Bolts slipping into bearing under service loads do not compromise the strength of the connection or the safety of the structure. Engineers who encounter this situation may be required to provide this information to the building owner and/or occupants to satisfy their concerns.

Calculations indicate that the building in question is probably more flexible than usual which could contribute to the susceptibility of connection slip. It is also Mr. Disque's opinion that the very thin flange cover plates could bend inperceptually and would not necessarily inhibit slips in the web connection.

References

Galambos, T. V., editor. (1988). *Guide to stability design criteria for metal structures*, 4th Edition, Wiley-Interscience, New York, N.Y

Wiesner, K. B. (1986). "Design office solutions for a 2-story beam column." *Proceedings, 1986, AISC National Engineering Conference*, Nashville, Tenn., June 12–14.

2.15 Panel Zones in Rigid Frames

What is the effect of panel zone deformation on frame drift?

Response.

Shear deformations within the panel zone (column web area at beam to column intersections) reduce the overall stiffness of a rigid frame and thus increase frame drift resulting from externally applied loads. When external loads are inertial forces, approximating a seismic incident, panel zone deformations affect frame drift more significantly than a loading condition that is not seismic (Tsai and Popov 1988).

Research conducted on panel zones of rigid frames (Tsai and Popov 1988; Krawinkler 1978; Kato et al. 1988) provides information on methods for modeling the stiffness of the panel zone to approximate increased frame drift. Shear force–shear distortion models of panel zones developed in the references by Tsai and Popov (1988) and Krawinkler (1978) can be used to generate moment-rotation relationships to be used in a drift analysis to reflect panel zone deformation effects on frame drift.

References

Kato, B., Chen, W. F., and Nakao, M. (1988). "Effects of joint-panel shear deformation on frames." *Journal of Constructional Steel Research*, Vol. 10, pages 269–320.

Krawinkler, H. (1978). "Shear in beam-column joints in seismic design of steel frames." *AISC Engineering Journal*, Vol. 15, (No. 3; Third Quarter); pages 82–90.

Tsai, K. C., and Popov, E. P. (1988). "Steel beam column joints in steel moment resisting frames." *Report No. UCB/EERC 88/19*, Earthquake Engineering Research Center, University of California, Berkeley, Calif., December, 1988.

2.16 Slip in Torqued Bearing Connections at Service Loads

There have been a number of recent instances reported of loud banging noises occurring during the construction (and sometimes early into the occupancy) of steel-framed buildings. In most cases, this banging is apparently the result of the slipping of bolted connections under service loads. What is the most likely cause of this banging, and does it have any potential ramifications relating to the safety of the structure?

Response.

A survey was performed of engineers and fabricators familiar with these occurrences to obtain information related to the cause of this condition. The following conclusions were drawn from this survey.

1. The connections that experienced this banging typically support composite beams with spans over 30 ft (18 m). This condition has been reported for both shored and unshored composite beam construction.

2. In all cases, the banging occurred at shear connections where the high strength bolts were designed using bearing values, but the bolts were fully torqued using tension control ("twist-off") bolts or the turn-of-the-nut method.
3. In most cases, single plate shear connections were used. There have been cases, though, of single-angle and double-angle connections that have experienced this banging.
4. Both standard size and short-slotted holes have been specified in various cases.
5. None of the cases reported the use of painted faying surfaces.
6. In most cases, the banging occurred during construction and fit-up of the steel frame. Some instances of banging have been reported early in the occupancy of structures, but no cases of this condition beyond the first year have occurred.
7. In *no* case has any type of failure or structural distress been reported to be related to this condition.

It appears, then, that the banging noises that result from the slip of high-strength bolted connections under service loads is mainly the result of the common practice of specifying fully torqued bolts in connections designed for bearing values (Section J1.9 in LRFD; Section J1.12 in ASD). The AISC *Specification* provides a list of connections for which fully tensioned high-strength bolts or welds shall be used. These connections include column splices and bracing in tall buildings, support structures for large cranes, conditions under which impact or stress reversal can occur, and "any other connections stipulated on the design plans." The section goes on to state that "in all other cases, connections may be made with high strength bolts tightened to the snug-tight condition or with A307 bolts." Engineers may, therefore, choose to allow snug-tight connections with high-strength bolts in conditions that could result in banging bolts.

The banging bolts question is strictly a serviceability issue. Bolts slipping into bearing under service loads do not compromise the strength of the connection or the safety of the structure. Engineers who encounter this situation may be required to provide this information to the building owner and/or occupants to satisfy their concerns.

References

Birkemoe, P. C. (1983). "High strength bolting: Recent research and design practice." *Proceedings of W. H. Munse Symposium on Behavior of Steel Structures—Research to Practice*, W. J. Hall and M. P. Gaus, Editors, ASCE, New York, N.Y.

Mann, A. P., and Morris, L. J. (1984). "Lack of fit in high strength connections." *Journal of Structural Engineering*, ASCE, Vol. 110, (No. 6; June), pages 1235–1252.

2.17 Tee-Type Simple Beam Shear Connections

Design guidelines for all-bolted and bolted-welded tee-type simple beam shear connections are needed.

Response.

Four types of tee-type shear connections are appropriate, as follows: (1) All bolted; (2) all welded; (3) bolted to beam web, welded to column; and

David T. Ricker, P.E. He is retired now, living in Payson, Arizona. Dave was the Chief Engineer of Berlin Steel at the time of this project, and he remembers the project. He believes the steel frame was fabricated and erected under tight supervision. He remembers the frame swaying a lot in the wind during construction. He is familiar with the "banging bolt" phenomena and believes this is the best explanation for this building.

Bill Hubble, Superintendent. He was the Superintendent for the General Contractor on the Annex. We know him from five projects we have worked on with him and have respect for his capability.

Cracks in stair walls have been there from day one. Bill had Jim Brockman out to look at stairs during construction. Said stairs were tied in and when steel frame moved, block cracked. Also he said that stairs were hung from the steel but they were made to cut the hangers and let the stairs bear on the block.

Note: This statement is consistent with existing evidence of paint inside the cracked surfaces.

Stephen Carey, Public Safety Department. He said that Dave Raffles under contract to DPW reviewed the structural design before construction. This was not a formal peer review because this occurred during the interim period when there was confusion as to whether peer review applied to State of Connecticut projects. The records on file do not indicate any problems with the design of this project.

Steve remembers Purcell Associates provided Construction Administration and that the DPS felt this project was receiving close attention and inspection.

Laurie Wood, Fusco Management Company. She is the on-site property manager, and she interviewed DEP staff members who were close to the noise source. She collected very consistent descriptions immediately after events, before people had a chance to exchange stories with each other.

Our interviews of DEP staff were consistent with her reports that pinpointed the locations. People below heard noise only, people for two floors above heard the noise and felt floor vibrations.

Jim Brockman, P.E., Partner in Macchi Engineers. They were Engineer of Record for the structural design of this project. He sent an engineer to investigate on January 10, 1994 when a loud noise was reported at 8:30 a.m. He also was called to investigate after the December 23 1998 event. Jim's opinion then and now was a car impacted a massive concrete column in the parking garage, causing the noise.

NOTICE OF ALLEGED SAFETY OR HEALTH HAZARDS

The following structural damages have appeared since this phenomena began:

1. The stairwells in the addition have encountered continuous running horizontal cracks which encircle the stairwell beginning at each floor.

   Comment: This was answered in this report.

2. Some doors in the addition are not aligned properly.

   Comment: Not related to noise problem.

3. There are vertical cracks above and below the windows on the west side of the building.

   Comment: Not related to noise problem.

4. On the second floor the west wall bows in some places.

   Comment: Measurements on interior do not confirm this.

5. On the third floor there is an area where the concrete floor has been slightly raised and appears to be cracked.

   Comment: This was unanswered in this report.

The Connecticut State Employees Association (CSEA) represents approximately 550 DEP employees. The majority of these employees are housed at 79 Elm Street. We have concerns regarding the structural integrity of the building and the safety of our member.

Comment: After conducting a structural investigation of the Annex, we have found no evidence of risk to the structural integrity of the building.

CONCLUSION

We know the building is laterally flexible in the short direction. The best explanation for the loud noise events of which there have been two or three reported that we know of, is “banging bolts”. This building fits all of the criteria except for the timing. Normally the noise occurs during construction or shortly after occupancy. In spite of that, we believe this is the cause of the noise. While this is not a safety issue, it could happen again.

We have conducted a reasonable level of investigation and have found no indication of unsafe structural conditions.